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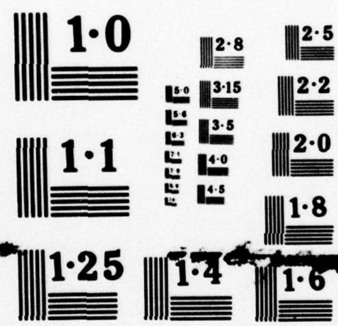

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THE DYNAMIC LOOP EFFECT ON THE MISSISSIPPI RIVER, PROJECT DESIGN FLOOD FLOW LINE

by

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Final Report

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→ The unusually high 1973 and 1975 flood seasons on the Mississippi River revealed apparent significant reductions in flood-carrying capacity. In addition, a heretofore unappreciated phenomenon known as loop effect appeared to have a large impact on stage-discharge relations. As a part of a general reanalysis of the design flood flow line, a study was initiated to help quantify the dynamic loop effect. The purpose of the study was to estimate the loop

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20. ABSTRACT (Continued).

magnitude at the design flood to be added to the separately evaluated steady-state design flood flow line.

A previously developed numerical model for calculations of variable stage-discharge relations was used. The study showed that this model could not account for all of the loop magnitude, probably due to flow complexity and model assumptions. An ad hoc method to simulate the total loop was implemented involving double-valued, variable Manning's n corresponding to the rising or falling phase of the flow hydrographs. The method of simulation suggests a higher channel resistance at falling river stages. Extrapolation to the design flood is based on the assumption of similarity of river behavior to the 1973 flood. This assumption is consistent with the steady flow line computations using river resistance extrapolation from the 1973 flood data to the design flood.

The modified model was applied to three representative gaging stations to quantify the loop magnitude along the lower river. Results of the study indicate that the dynamic loop allowance should be as follows:

Vicksburg	- 0.9 ft
Helena	- 1.0 ft
Baton Rouge	- 0.5 ft

In summary, an allowance of approximately 1.0 ft should be added to the flow line from about Helena to Red River Landing. An amount of 0.5 ft should be sufficient from Red River Landing to about Baton Rouge.

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PREFACE

This investigation was sponsored by the U. S. Army Engineer Division, Lower Mississippi Valley (LMVD), and funded jointly by the U. S. Army Engineer Districts, Memphis, Vicksburg, and New Orleans. The study was conducted during the period March 1976 to October 1978 by the Hydraulics Laboratory of the U. S. Army Engineer Waterways Experiment Station (WES) under the general supervision of Messrs. H. B. Simmons, Chief of the Hydraulics Laboratory, and M. B. Boyd, Chief of the Hydraulic Analysis Division. A preliminary report presenting final computational results at the Vicksburg and Helena gaging stations and tentative results at Baton Rouge was presented to LMVD in May 1977.

The study was conducted and this report was prepared by Mr. Carl Huval. Mr. M. B. Boyd contributed substantial technical input to the study. Special assistance in providing data was contributed by Mr. Malcolm Dove, LMVD, as well as personnel from the Vicksburg, Memphis, and New Orleans Districts.

Commander and Director of WES during the investigation and the preparation and publication of this report was COL John L. Cannon, CE. Technical Director was Mr. F. R. Brown.

CONTENTS

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
cubic feet per second	0.02831685	cubic metres per second
feet	0.3048	metres
feet per second	0.3048	metres per second
feet per second per second	0.3048	metres per second per second
miles (U. S. statute)	1.609344	kilometres
square feet	0.09290304	square metres

THE DYNAMIC LOOP EFFECT ON THE MISSISSIPPI RIVER PROJECT DESIGN FLOOD FLOW LINE

PART I: INTRODUCTION

The Problem

1. Recent large-scale flooding during the 1973 and 1975 flood seasons on the Mississippi River and its tributaries indicated that apparently significant reductions had occurred in the flood-carrying capacity of the river in some reaches.[1] These changes resulted in higher stages for given discharges and could have a major impact on the degree of flood protection provided by engineering works such as levees. As a result of the 1973 flood, a reanalysis of the project design flood flow line was initiated by the U. S. Army Engineer Division, Lower Mississippi Valley (LMVD), and the U. S. Army Engineer Districts, New Orleans, Vicksburg, and Memphis. The main effort of the district studies involved the use of a steady-state flow model, HEC-2, Water Surface Profiles,[2] to recompute the project flood flow line along the river. The 1973 flood peak stage and discharge data and recent channel cross-section measurements were used to recalibrate and then extrapolate to design flows.

2. In addition to apparent reduction in channel capacity, discharge measurements indicated that some increase in stage was attributable to loop effect. It is well known[3] that changing discharge during a flood can produce a so-called "hysteresis loop" in the stage-discharge rating curve such as the one shown in Figure 1. For this type of rating curve, there can be several different stages for a given discharge. The lower stage value is associated with the rising limb of the discharge hydrograph and the higher value occurs during the recession of the discharge. A hysteresis loop, which is caused by a variable energy slope due to changing discharge, has also been referred to as a "dynamic loop" since it occurs because of the changing or dynamic nature of the flood

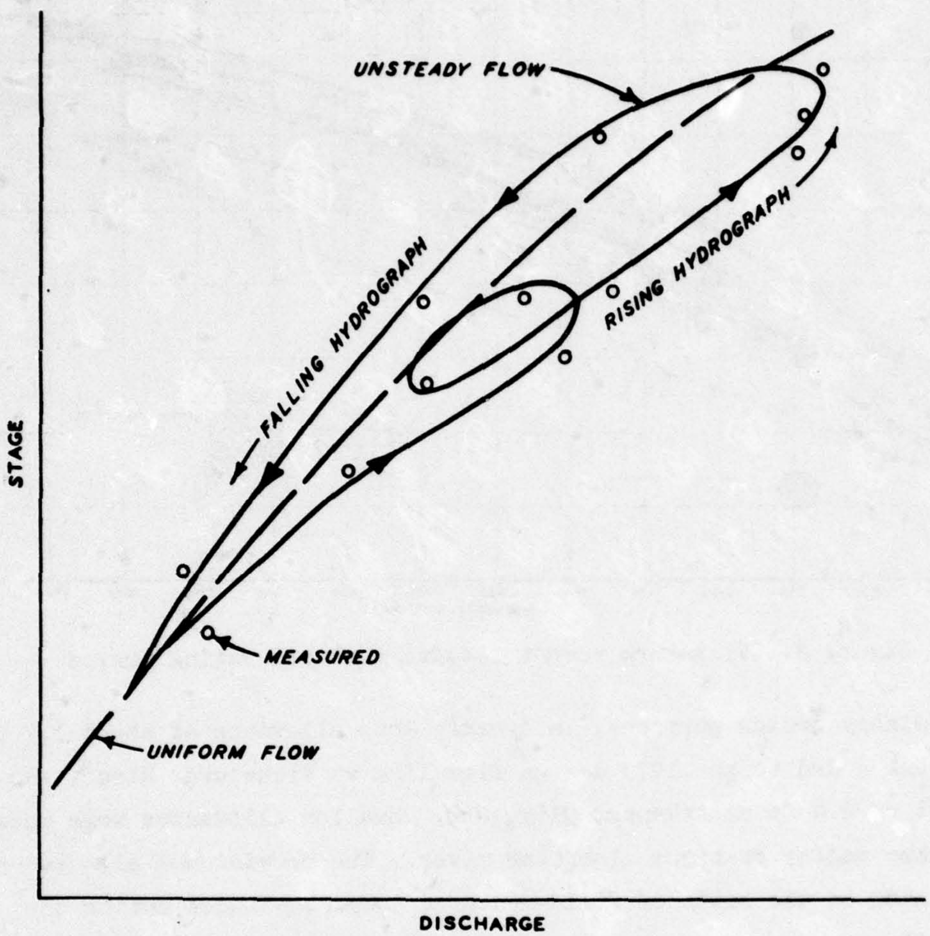

Figure 1. Typical loop rating curve

discharge. A plot of several recent loop rating curves at Vicksburg is shown in Figure 2. Prior to the 1973 flood, the 1950 flood was the most recent major flood event on the Mississippi River. As shown in Figure 2, an adopted rating curve had been used for design based on the 1950 flood. The 1973, 1974, and 1975 flood rating curves are significantly higher than the one for 1950. As shown in Figure 2, the stage-discharge relation can be very complex, since it is dependent on several river and inflow factors.

3. The 1973 flood showed that the project design flood flow line had to account for the looping effect on the Mississippi River. For

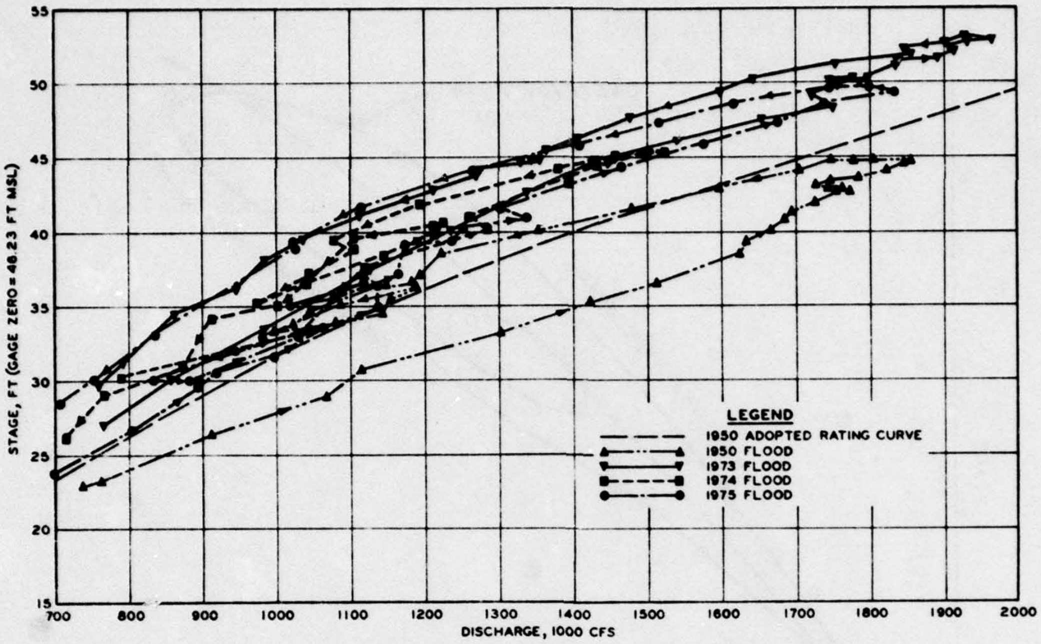

Figure 2. Vicksburg recent floods, measured rating curves

preliminary design purposes,[1] a dynamic loop allowance of about 1.7 ft* had been added to the 1973 design flow line at Vicksburg, Miss., and an amount of 2.4 ft at Arkansas City, Ark. Smaller allowances were made at other gaging stations along the river. The provisional allowances were made on the basis of field-measured loop magnitudes during the 1973 flood and a study of past rating curves along the river. A flow line review study requirement by the LMVD was to use an analytical and hydrologically consistent method for computation of the dynamic loop effect.

Purpose and Scope of Study

4. The purpose of this study is to identify dynamic loop and other unsteady flow effects during the 1973 flood and to estimate the loop

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* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 3.

magnitude at the design flood. The loop magnitude is the amount to be added to the steady-state project design flood flow line computed using HEC-2 at the design flood. As a part of the study, the model developed by Fread[4] was implemented for use in providing a consistent and improved calculation method for determining dynamic loop effect for adjusting the steady-state flow line. The 1973 flood year was used as the basic calibration and Vicksburg, Miss., Helena, Ark., and Baton Rouge, La., gaging stations were investigated. The model was calibrated against the 1973 flood at each of those stations and computations were made for the project design flood 58A-EN. In order to determine the effects of consecutive loops occurring on the rising limb of the flood hydrograph, several discharge hydrographs were run to simulate other potential flood hydrographs. The two hydrographs described below were selected as the main input flows for the loop computations: (a) combined hydrograph using the top part of the hypothetical project flood hydrograph 58A-EN with the observed rising part of the 1973 flood hydrograph and (b) the hypothetical project flood hydrograph 58A-EN alone, which has one single peak.

PART II: THE MODEL

Background

5. The river engineer is frequently concerned with the conversion of flood stages at a given location along a stream channel into corresponding discharges or vice versa. This is accomplished by a relation between stage and discharge which applies to that particular gaging station. Such a relation or "rating curve" is usually developed empirically from a number of previous streamflow measurements and the corresponding stages. Unfortunately, the observed measurements of stage and discharge will not usually form a unique relation, i.e., a single value of stage does not correspond to a single value of discharge. Deviations of the measurements from a single-valued rating curve can be the result of such factors as:

a. Discharge measurement errors.

b. Shifting control due to scour or fill and alluvial bed form changes.

c. Variable water-surface slope along the channel due to unsteady flows.

d. Local effects causing variable water-surface slope due to backwater from a tributary or overbank storage.

e. Water temperature effect on alluvial bed movement.

6. While the effects due to errors, sediment movement, and tributary and overbank flows are not easily quantified, flow accelerations due to unsteady nonuniform flow can be computed. In a recent report, Fread[4] presented a relatively simple mathematical model that retains most of the dynamic loop effects. The method was proved accurate for calculating the dynamic stage-discharge relation at a number of river gaging stations.

Model Details

7. A thorough description of the model with the computer program documentation has been presented by Fread.[4] The basic concept is to

model the stage-discharge relation for a particular gaging station location on the river, i.e., equation terms involving spatial gradients of discharge and flow depth are transformed into time variations at the location. The model development requires several assumptions that are applicable for a short section of channel containing the gaging station or forecast point:

- a. Lateral inflow or outflow is negligible.
- b. Channel width is essentially constant, i.e., $\partial B/\partial x \approx 0$.
- c. Energy losses from channel friction and turbulence are described by the Manning equation.
- d. Geometry of the section is essentially permanent, i.e., any scour or fill is negligible.
- e. Bulk of the flood wave is moving approximately as a kinematic wave which implies that the energy slope is approximately equal to the channel bottom slope.
- f. Flow at the section is controlled by the channel geometry, friction, and bottom slope and by the shape of the flood wave.

8. If the bulk of the flood wave moves approximately as a kinematic wave, the following expression can be used to transform the spatial gradient to time variations of flow depths:

$$\frac{\partial y}{\partial x} = -\frac{1}{c}\frac{\partial h}{\partial t} - \frac{2}{3}\frac{S_o}{r^2}$$

where

- y = depth of flow, ft
- x = distance along the river channel, ft
- c = kinematic wave velocity, fps
- h = water-surface elevation above mean sea level, ft
- t = time, sec
- S_o = effective bottom slope of the channel
- r = ratio of the channel bottom slope to an average flood wave slope

The last term on the right of the equation is a correction factor and is usually small for large river applications. The kinematic wave velocity c is computed from the Kleitz-Seddon law.

$$c = \frac{1}{B}\frac{dQ}{dh}$$

where

B = channel width at the water surface, ft

Q = discharge, cfs

If the channel is wide and prismatic and flow can be determined by Manning's equation, the wave velocity is

$$c = K\frac{Q}{A}$$

where $K = 5/3 - (2A/3B^2)(dB/dh)$. In the above development, the hydraulic radius R is approximated by the hydraulic depth D . The result of the above expression for wave velocity is that the propagation speed of the flood wave in the river is a multiple of the river mean velocity. The quantity K varies from 1.3 to 1.7 for triangular- to rectangular-shaped channels, respectively. For a given channel geometry, the speed of wave propagation varies uniquely with discharge and stage. The wave speed is assumed the same for the rising and falling portion of the input hydrograph. Overbank storage or flows, backwater areas, tributary effects, and other dynamic phenomena are not accounted for in the model and can become important with complex river systems.

9. With the definition of the wave speed c given above and the method shown by Fread[4] to evaluate r , it is possible to obtain the following equation relating stage and discharge:

$$Q = 1.486\frac{AD^{2/3}}{n}\left\{S_o + \left[\frac{A}{KQ} + \left(1 - \frac{1}{K}\right)\frac{BQ}{gA^2}\right]\delta h_s + \frac{Q'/A' - Q/A}{g\Delta t} + \frac{2S_o}{3r^2}\left(1 - \frac{BQ^2}{gA^3}\right)\right\}^{1/2}$$

where

δh_s = change in water-surface elevation during the time interval Δt , in fps ($\delta h_s = (h - h')/\Delta t$, where h' is the stage at time $t - \Delta t$).

Q' = discharge at time $t - \Delta t$, cfs

A' = cross-sectional area at time $t - \Delta t$, sq ft

g = acceleration due to gravity

Δt = small interval of time, sec

The above equation is utilized in the model as a basis for computing discharge when the rate of change of stage is known or stage when the rate of change of discharge is known. The unknowns Q and h in the above equation are not expressed in an explicit manner, thus requiring a trial-and-correct solution. The solution of the equation is by Newton iteration and is explained fully by Fread.[4]

Model Implementation

10. The model (called DYNMOD by Fread) had been implemented by LMVD on the WES Honeywell 635 computer by the time-sharing system. Several modifications had been introduced for ease of application. The model requires the input of either a discharge or stage hydrograph and will compute the other hydrograph. In this application, stage hydrographs were input for calibration against field-measured stage-discharge relations at the gaging stations. The model was then used in the discharge input mode to study loop rating curves using several design discharge hydrographs.

11. Channel input geometric data consist of: (a) the channel effective bottom slope (S_o), (b) the cross-sectional area (A) and the surface width (B) as a function of the elevation, and (c) the Manning's coefficient (n) as a function of the elevation. These are input into the model in the form of data tables at about 10 elevation intervals. The area and width were obtained from the tabulated 1973 and 1975 discharge observation data and measured cross sections at the gaging stations. Plate 1 shows the cross section at the Vicksburg gaging site

during the 1973 and 1974 high-water hydrographic surveys. A low-water survey for 1971 is also shown for comparison. Plotted channel cross-sectional area and widths for the 1973 and 1975 flood years are shown in Plate 2 for the Vicksburg gaging section. The graphical method of extrapolating to design stage levels is also shown. A preliminary estimate of the effective slope was obtained from the average low-water plane, 1973 peak flood data, and channel bottom slope at the gaging stations. Trial computations required adjustment of this value to give a better representation of the loop rating curves. The Manning's n values were calculated by fitting single-line rating curves at the gaging stations using Manning's equation and known channel slope and geometry. Several combinations of slope and resulting n values were tried.

PART III: MODEL CALIBRATION AND APPLICATION

Vicksburg, Miss.

12. After model familiarization, the first step was to recompute a published test case given in the basic reference by Fread.[4] This provided a check to ensure proper function of the computer code. Several improvements to the model were developed during the study. The data input and output structure was revised to permit the use of files for ease of data manipulation. A plotting capability using an available Hewlett-Packard plotter was developed for easier interpretation of the output information.

13. Most of the model testing and modifications described below were made during the calibration phase at the Vicksburg, Miss., gaging station. Several combinations of river slopes and channel roughness were tried. The river slope at Vicksburg is about 0.000064 ft/ft. Computations were made at effective slopes from 0.000064 to 0.00001 and comparisons were made with the measured stage-discharge data. The smaller slopes tend to increase the dynamic loop width, but also tend to amplify daily stage variations. A compromise is thus required to give reasonable model response for reproduction of the 1973 rating curve. A value of 0.000032 was finally selected. Table 1 gives a summary of the geometric input and channel roughness data at Vicksburg. The data tabulated are representative of the main channel only. The plot in Plate 3 shows the resulting computed and measured stage-discharge curves at Vicksburg; the measured channel discharge plotted has been corrected for the overbank flow. It was apparent from these results that the basic DYNMOD model would not reproduce the magnitude of the loops as compared with the measured 1973 stage-discharge curve. The dynamic loop effect at lower discharges for within bank-full rising and falling stages is reproduced very well but the computed main loop is only about one half the observed main loop.

14. The model DYNMOD was modified by using different Manning's n roughness values for rising and falling periods of the flood. The

basic model uses a single set of n values which along with other input data define a mean rating curve. The difference between the computed dynamic stage-discharge relation and the steady-flow curve represents the estimate of the dynamic terms. Since the results shown in Plate 3 suggest that the dynamic terms are not large enough to account for the observed loop, a variable set of n values was used to obtain better agreement with the observed 1973 flood data. In effect, the different n values for rise and fall were used to account for several factors which cannot be properly evaluated and modeled by DYNMOD, such as channel geometric changes from scour, changes in effective resistance coefficients due to changing bed forms, interactions with overbank storage and flow, backwater storage, and inaccuracy of estimates of the dynamic terms.

15. Manning's n values to be used for rising and falling portions of the flood were developed by drawing two rating curves through the 1973 flood data (Plate 3) and computing the corresponding n values (Table 1). The rating curves were extrapolated to stages slightly above the design stage to provide roughness values for that stage range (Plate 5). After modifying the DYNMOD model to permit use of the different n values for rise and fall, a stage-discharge curve was again computed for the 1973 flood. There was substantial improvement in agreement for the main loop but the within bank-full loop was distorted. The computation tended to switch between the two sets of n values at each rise or fall of the stage hydrograph. The model was then further modified to allow the introduction of a switch to be used to control the model to use the rising or falling set of n values, depending on predetermined values of stage and discharge. This modification allows the use of the rising n values up to a preset elevation or discharge, switches to the falling n values on the first fall which occurs after the hydrograph passes through the reference elevation or discharge, and retains the use of the falling n values for the remainder of the computation even if another rise occurs. Using this procedure and a reference elevation of 97 ft msl (about 50.8 stage at Vicksburg), the computed stage-discharge curve for the 1973 flood compared favorably with the

observed curve (Plate 4) and the model was considered calibrated for further use at the Vicksburg gaging station.

16. The design discharge at Vicksburg for the 58A-EN project design flood (PDF) is 2,710,000 cfs. The peak flow during the 1973 flood was 1,960,000 cfs which is about 75 percent of the PDF. In order to study loop effects at design flows, an extrapolation of channel geometry and roughness was required. Plate 2 shows the method of extrapolation. Channel area and width were extrapolated from the 1973 and 1975 data plots, taking into consideration a plot of the river cross section. Overbank areas and widths were eliminated by vertical extension of the main channel above bank-full. The areas and widths used in the computations above bank-full flow were representative of the main channel only, which was necessary because of model limitations.

17. An initial trial computation was made by extrapolating the 1973 rating curve at Vicksburg to design flows. Two rating curves representative of the rising and falling part of the hydrograph were extrapolated graphically. After additional study, it was decided that calibrated channel Manning's values from the 1973 flood would be extrapolated to design flows. Plate 5 shows a plot of channel roughness variation with stage and the extrapolation for the rising and falling part of the hydrograph. Overbank flows during the 1973 flood were subtracted from the total discharge. The PDF hydrograph was also modified to account for estimated overbank flows at design discharges. Overbank flow was subtracted from the total design flow, based on steady-flow conveyance computations furnished by the Vicksburg District. The method of flow partition was based on computations made using the steady-flow backwater computations using HEC-2. The peak flow at Vicksburg, for example, was reduced from 2.71 million cfs to 2.56 million cfs. The resulting design hydrographs are tabulated in Table 2.

18. After the calibration and modifications previously discussed had been made to DYNMOD, two computations were made using as input the project design flow hydrograph 58A-EN and a composite hydrograph which follows the 1973 flood through its peak flow of about 1.96 million cfs, then assumes a subsequent rise to the design flow of 2.56 million cfs

following the shape of the 58A-EN hypothetical flood (Table 2). Results of the computations are shown in Plate 6. The double-valued Manning's n computation was used to simulate the effects of a 1973 type of flood followed by the design flood, using the falling n values for flows above the 1973 peak and up to design discharges. The run using only the rising n values and the 58A-EN flood simulates the Vicksburg rating curve if the channel roughness had not increased for corresponding stages during the design flood. The difference between the two curves at design flows is a measure of the dynamic loop effect (plus other unaccounted for factors such as overbank effects and bedform changes). Assuming that the higher channel roughness would be appropriate if the design hydrograph followed the 1973 peak flows, the difference in the two curves can be used for estimating loop effect. The computations indicate a difference of about 1.7 ft in the peak stages with and without the loop effect at Vicksburg.

Helena, Ark.

19. A plot of the Helena discharge range cross section is shown in Plate 7 for the 1973 and 1975 high-water surveys. The 1973 cross section was used to calculate the cross-sectional area and channel width and these values are plotted in Plate 8. Other width and area measurements made in 1973 and 1975 flood years at the discharge range are also plotted in Plate 8.

20. A calibration procedure was used at Helena similar to that developed for the Vicksburg gaging station. The final results of several trial computations at Helena are shown in Plate 9. The switch from rising to falling n values was set to 48.6-ft stage and 1,550,000 cfs for Helena to provide the best fit to the 1973 flood discharge data. The plot indicates good agreement with the channel discharge measurements. The channel geometric data and Manning's n computations are presented in Table 3. The cross-sectional area and width were based on the average data plotted in Plate 8. Channel area and width at design stage were obtained by plot extrapolation. The calculated set of

Manning's n is shown plotted in Plate 10. Several channel slopes were tried from the apparent channel slope of 0.00011 down to a value of 0.000032 ft/ft; a slope of 0.000064 was finally selected. The effective channel slope and computed roughness values at Helena are about twice those at the Vicksburg gaging station. The channel roughness values at Helena appear to be in agreement with other recent studies[5,6] of Mississippi River channel roughness. The Vicksburg values seem to be low. Increasing the slope at Vicksburg to 0.000064 would increase n values by $\sqrt{2}$ or 41 percent, which would still be lower than the Helena data. Several factors not accounted for in the model, such as Yazoo backwater area effects and changing river sinuosity with stage, could be the reason for these apparent discrepancies.

21. Computations were run using the 58A-EN design flood at Helena and a design discharge hydrograph made up of 1973 daily Helena discharges phased into the 58A-EN hypothetical flood at Helena. These data are tabulated in Table 4. The design hydrograph was corrected for overbank flows based on backwater computations by the Memphis District. One run was made using the 1973/58A-EN hydrograph with different Manning's n values on the rising and falling parts of the hydrographs. Plate 11 presents the results. Similar reasoning at Helena to that developed at Vicksburg relative to the sequence of flood flows and channel roughness suggests that the difference between the two curves at design flows is a measure of the dynamic loop and other effects. The difference in the two curves can be used for estimating loop effect and indicates the total loop effect to be about 2.0 ft at Helena.

Baton Rouge, La.

22. The channel cross section at the Baton Rouge gaging range is plotted in Plate 12 based on the January 1974 hydrographic survey. There is no overbank area at Baton Rouge. The calibration procedure was modified at the Baton Rouge gaging station. A study of the 1973 data indicated large scatter in the channel cross-sectional area and width with stage (see Plate 13). During the 1973 flood, discharge measurements

were made only during rising stages from 39 ft to 42 ft and falling stages to about 32 ft. The loop rating curve is very hard to define and extrapolate to design flows without discharge data to show the rising and falling hydrographs at a wide range of river stages. Additional data were reviewed from the 1974 and 1975 high water but these showed similar data scatter and a narrow range of river stage. In addition, measurement techniques and discharge range location apparently were varied during 1974 and 1975.

23. Area and width data from the 1950 flood were also plotted in Plate 13 and showed much less scatter. Discharge measurements were made over a wider range of stages during the 1950 flood. The 1950 discharge range was located 3.1 miles downstream from the 1973 discharge range which was at the U. S. Highway 190 Bridge or river mile 230.8, 1962 AHP. It was decided to calibrate the model using both 1950 and 1973 flood years. The channel data for the 1950 discharge measurements appeared adequate. A study of the 1973 discharge data indicated that most of the measurements were made at a discharge range about 200 ft downstream of U. S. Highway 190 Bridge. The channel cross section shown in Plate 12 was used as the best source of geometric information for the 1973 flood. Table 5 presents the adopted geometric data for the 1950 and 1973 floods. Note that Manning's n values were not varied for the rising and falling portion of the hydrograph. As suggested above, the basic data did not warrant the use of multivalued Manning's n coefficients. As had been done previously at Vicksburg and Helena gaging stations, several channel slopes were tried from 0.00001 to 0.000032 ft/ft. The value of 0.000016 ft/ft was used.

24. The calibrations of the model to the 1950 and 1973 measured discharge data are shown in Plates 14 and 15, respectively. The 1950 calibration gave poor reproduction of the rising part of the hydrograph up to about the 41-ft stage. Agreement was adequate at higher stages. The 1973 calibration showed large fluctuations and very complex stage discharge relations at stages above 39 ft. It is speculated that gate operation at the Morganza Floodway caused these variations. The resulting Manning's n values are plotted in Plate 16.

25. The project design flood hydrograph 58A-EN at Baton Rouge was furnished by the New Orleans District and is shown in Table 6. Because of the designed operation of the Morganza Floodway during a large flood, flows at Baton Rouge would be kept below 1,500,000 cfs. In order to provide a realistic hydrograph that gives some weight to flow sequences, a composite design hydrograph was obtained at Baton Rouge. This hydrograph was calculated by combining the 58A-EN hydrograph with increased 1973 daily flows from April 18 through June 30. The daily flows at Baton Rouge were multiplied by the ratio of 1,500,000 to 1,347,000 (1.114) assuming the design flow maximum of 1,500,000 cfs occurred on 18 April 1973. This was the first discharge measurement available at Baton Rouge after the opening of the Morganza Floodway on 17 April 1973. This hydrograph is an attempt to simulate the river behavior at Baton Rouge which is representative of 1973 conditions at the design flow of 1,500,000 cfs. Both design hydrographs are shown in Table 6.

26. Computations were made with both design hydrographs and are shown in Plates 17 and 18. The design flood 58A-EN computations show a dynamic loop effect of about 0.7 ft near the peak flow and a maximum stage of 45 ft. The composite 1973/58A-EN design flood computations result in a loop effect somewhat less than 1.0 ft and a maximum stage of 45.2 ft. Results of both computations indicate a dynamic loop effect of about 1.0 ft at Baton Rouge. The amount of dynamic effect in the Mississippi River in the Old River to Baton Rouge reach during a design flood will be greatly dependent on the mode of operation of the Old River Control Structure and Morganza Floodway. If these operations are expected to be greatly different than design or 1973 conditions, dynamic effects from gate manipulations may cause higher stages. A more thorough analysis using a complete dynamic model would be needed to ascertain stage response to variable gate structure operations.

PART IV: DISCUSSION OF RESULTS

27. For high flows at the Vicksburg and Helena gaging stations, the computation follows the extrapolated n values for either the rising or falling portions of the 1973 flood (Plates 6 and 11). The difference between computations for the different flood sequences is attributed to loop effect. The rationale supporting this type of projection is: had the last rise to design flow occurred in 1973 or a similar antecedent flood, the roughness values compatible with the falling portion of the antecedent flood would be more appropriate during that rise than the rising n values associated with the lower channel roughness. The use of the modified Fread model DYNMOD in this fashion is <u>to provide for a judgment prediction of the loop effect rather than as a precise predictive or modeling tool</u>. Nevertheless, the loop adjustment suggested by these projections is believed to be valuable in supplementing the steady-state flow line computed from HEC-2. In other words, the dynamic model was used in the analogous manner as steady-state computations were made and the model is believed consistent and comparable in accuracy.

28. The setting of the project design flow line is to be used for the design of levee crest heights and other water-control structures along the main stem of the Mississippi River. Previous flow line computations in 1957 had been made using steady-state backwater computations and maximum design discharges. The rating curves used were a series of single-valued curves based on field measurements during the 1945 and 1950 floods. This type of computation was repeated using the 1973 flood data as a part of the flow line reanalysis. The previous LMVD report[1] recognized the need to allow for dynamic effects over and above the steady-state flow line. Steady-state (or equivalently, single line) rating curves are usually an average between the rising and falling part of the hydrographs. A loop magnitude allowance that would be consistent with the steady-state flow line would then be about one half the total dynamic loop magnitude.

29. Results obtained with the DYNMOD model suggest that channel dynamic effects are small near design flow conditions even though they

may be substantial at smaller flows. Even at the lower flows, dynamic effects do not appear to account for the entire loop observed during the 1973 flood. With the model used herein, it was necessary to lump the effects of several factors into the roughness coefficient. This emphasizes the need for accuracy in extrapolating roughness coefficients to design flows and for investigations to evaluate the effects of other complex river factors. Better definition of the effects of some of these factors can be obtained with more detailed studies using modeling methodologies such as developed for the junction near Cairo, Ill.[7] Fread has recently suggested a methodology for more accurate treatment of the overbank effect[8] that includes overbank storage and flow between main channel meanders. Evaluating the effects of changing channel geometry and roughness due to bed changes will be more difficult. Because of the difficulty in predicting bed changes and their effects, it would appear advisable to use available modeling methodology to accurately evaluate all factors associated with a fixed geometry system before concluding that the sediment transport phenomena must be treated to obtain predictions of acceptable accuracy.

PART V: CONCLUSIONS

30. As a result of the investigation reported, the following conclusions have been formulated:

- a. The study of loop rating curves on the Mississippi River has shown that the Fread model DYNMOD will account for about one half of the total loop magnitude when using only a single set of n values.
- b. With computations using a single set of n values, peak design discharges produced the same maximum stage regardless of the shape of the discharge hydrograph, indicating that the dynamic factors alone do not properly account for total loop effect.
- c. The sequence of hydrograph flows does not have an appreciable effect on the rating curve predicted by DYNMOD using a single set of n values. The dynamic loop effect on the rating curve is small at and near design flows where rates of rise are usually small.
- d. The introduction of double-valued Manning's n for rising and falling hydrographs with a preset switch is necessary for simulating the 1973 flood event or other multiple peaked flood events.
- e. Using the modified model DYNMOD, it is possible to simulate dynamic effects at design flows, assuming that the loop behavior during the 1973 flood could have occurred at design flows.
- f. The rating curves computed assuming single-valued roughness with stage when compared with the rating curves computed assuming double-valued roughness give a reasonable extrapolation of the full dynamic loop effect. Several hydraulic phenomena such as tributary flows, channel-overbank interaction, and channel bed changes are lumped into the roughness values used in the model to simulate the 1973 flood.
- g. Considering the method in which the steady-state model calibration was accomplished, and the computational procedure used in the modified model DYNMOD, it is concluded that one half of the total loop effect should be used as the allowance for loop effect in establishing levee grades throughout the studied reach. Averaging the values calculated at Vicksburg and Helena, and using one half of the average, the total loop effect to be added to steady-state stages would be 1.0 ft.

h. An allowance of approximately 1.0 ft should be added to the flow line from about Helena to Red River Landing. An amount of 0.5 ft should be sufficient from Red River Landing to about Baton Rouge.

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3. Henderson, F. N., Open Channel Flow, McMillan Co., 1966, New York, N. Y.

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6. Robbins, L. G., "Roughness Characteristics of the Lower Mississippi," Symposium on Inland Waterways for Navigation, Flood Control, and Water Diversions--RIVERS 76, Aug 1976, Published by American Society of Civil Engineers, Fort Collins, Colo.

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8. Fread, D. L., "Flood Routing in Meandering Rivers with Flood Plains," Symposium on Inland Waterways for Navigation Flood Control and Water Diversions--RIVERS 76, Published by American Society of Civil Engineers, Aug 1976, Fort Collins, Colo.

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Table 1

Input Channel Data, Vicksburg, Miss.

Bottom Slope = 0.000032 ft/ft

Stage ft	Elevation ft msl	Area × 1000 sq ft	Width ft	Manning's n Rising Hydrograph	Manning's n Falling Hydrograph
13.77	60.0	105	2250	0.0286	0.0309
19.77	66.0	121	2480	0.0256	0.0272
29.77	76.0	145	2820	0.0202	0.0226
39.77	86.0	169	3120	0.0168	0.0186
44.77	91.0	183	3230	0.0159	0.0172
49.77	96.0	199	3300	0.0152	0.0163
53.77	100.0	220	3320	0.0148	0.0158
58.77	105.0	240	3320	0.0144	0.0153
63.77	110.0	262	3320	0.0141	0.0149

Table 2

Design Flood Hydrographs, Vicksburg, Miss.

Daily Discharge, 1000 cfs

Project Design Flood (PDF) 58A-EN			PDF 58A-EN Corrected for Overbank			1973/58A-EN				
520	2140	2670	520	2061	2525	500	842	1755	1962	2551
565	2175	2660	565	2092	2516	550	790	1770	1998	2542
630	2205	2650	630	2119	2508	600	747	1783	2029	2534
695	2230	2640	695	2141	2499	650	727	1780	2061	2525
750	2255	2630	750	2163	2491	700	721	1779	2092	2516
795	2275	2620	795	2181	2482	750	720	1779	2119	2508
825	2295	2605	825	2198	2469	800	761	1780	2141	2499
865	2315	2590	865	2215	2456	850	790	1784	2163	2491
900	2330	2575	900	2229	2443	900	823	1790	2181	2482
940	2345	2560	940	2242	2430	950	853	1796	2198	2469
980	2360	2540	980	2256	2413	1000	921	1797	2215	2456
1025	2375	2520	1024	2269	2396	1050	937	1799	2229	2443
1070	2385	2500	1067	2278	2378	1100	1048	1793	2242	2430
1110	2400	2480	1106	2291	2361	1128	1126	1788	2256	2413
1145	2415	2460	1140	2304	2343	1130	1158	1783	2269	2396
1180	2430	2435	1173	2317	2321	1145	1190	1760	2278	2378
1220	2450	2410	1211	2335	2300	1173	1234	1745	2291	2361
1265	2475	2385	1254	2356	2278	1184	1253	1760	2304	2343
1315	2505	2360	1301	2382	2256	1198	1280	1792	2317	2321
1370	2530	2330	1354	2404	2229	1209	1307	1810	2335	2300
1425	2555	2300	1405	2426	2203	1216	1337	1822	2356	2278
1475	2575	2265	1452	2443	2172	1222	1365	1836	2382	2256
1530	2595	2230	1503	2460	2141	1226	1394	1842	2404	2229
1585	2615	2190	1555	2478	2105	1226	1432	1851	2426	2203
1640	2635	2140	1605	2495	2061	1221	1465	1870	2443	2172
1695	2655	2085	1657	2512	2011	1206	1510	1892	2460	2141
1750	2675	2025	1707	2529	1958	1194	1556	1897	2478	2105
1805	2690	1960	1758	2542	1899	1180	1600	1904	2495	2061
1855	2700		1803	2551		1160	1654	1911	2512	2011
1900	2705		1844	2555		1138	1670	1909	2529	1958
1945	2705		1885	2555		1113	1688	1907	2542	1899
1990	2705		1926	2555		1060	1703	1916	2551	
2030	2700		1962	2551		1005	1720	1930	2555	
2070	2690		1998	2542		945	1731	1940	2555	
2105	2680		2029	2534		898	1742	1952	2555	

Table 3

Input Channel Data, Helena, Ark.

Bottom Slope = 0.000064 ft/ft

				Manning's n	
Stage ft	Elevation ft msl	Area × 1000 sq ft	Width ft	Rising Hydrograph	Falling Hydrograph
15	156.70	109	2620	0.0502	0.0556
20	161.70	122	2670	0.0412	0.0475
30	171.70	149	2800	0.0330	0.0363
35	176.70	163	2880	0.0295	--
40	181.70	178	2970	0.0287	0.0306
45	186.70	195	3150	0.0277	0.0297
51	192.70	220	3170	0.0260	0.0281
62	203.70	260	3170	0.0250	0.0268
65	206.70	272	3170	0.0246	0.0262

Table 4

Design Flood Hydrographs, Helena, Ark.

Daily Discharge, 1000 cfs

Project Design Flood (PDF) 58A-EN			PDF 58A-EN Corrected for Overbank			1973/58A-EN			
440	2290	1965	440	2101	1823	400	499	1849	2015
515	2305	1960	515	2114	1819	500	509	1905	1977
575	2320	1960	575	2126	1819	600	517	1956	1939
620	2335	1960	620	2139	1819	700	541	1998	1900
670	2345	1955	670	2148	1815	800	600	2033	1870
715	2355	1945	715	2156	1806	900	707	2058	1849
765	2365	1930	765	2165	1794	913	796	2084	1832
815	2370	1915	815	2169	1781	908	875	2101	1823
870	2375	1895	870	2173	1764	911	944	2114	1819
915	2380	1875	915	2178	1747	933	1002	2126	1819
965	2385	1855	965	2182	1730	951	1056	2139	1819
1005	2390	1835	1004	2186	1713	967	1086	2148	1815
1035	2400	1815	1030	2195	1695	987	1110	2156	1806
1060	2410	1795	1051	2203	1678	1004	1136	2165	1794
1095	2420	1770	1081	2212	1657	1023	1164	2169	1781
1130	2435	1740	1111	2225	1631	1044	1189	2173	1764
1170	2445	1705	1145	2233	1602	1052	1215	2178	1747
1215	2455	1665	1183	2242	1567	1048	1259	2182	1730
1260	2460	1620	1222	2246	1529	1033	1308	2186	1713
1310	2460	1575	1265	2246	1491	1018	1350	2195	1695
1365	2450	1530	1311	2237	1452	1006	1391	2203	1678
1425	2435	1480	1363	2225	1410	998	1425	2212	1657
1495	2415	1425	1422	2207	1363	989	1438	2225	1631
1575	2390	1370	1491	2186	1316	974	1449	2233	1602
1665	2360	1315	1567	2160	1269	949	1478	2242	1567
1755	2320	1270	1644	2126	1230	910	1509	2246	1529
1840	2280	1225	1717	2092	1192	845	1537	2246	1491
1920	2235	1180	1785	2054	1154	773	1560	2237	1452
1995	2190		1849	2015		691	1571	2225	1410
2060	2145		1905	1977		616	1580	2207	1363
2120	2100		1956	1939		570	1600	2186	1316
2170	2055		1998	1900		542	1620	2160	1269
2210	2020		2033	1870		521	1644	2126	1230
2240	1995		2058	1849		503	1717	2092	1192
2270	1975		2084	1832		497	1785	2054	1154

Table 5

Input Channel Data, Baton Rouge, La.

Bottom Slope = 0.000016 ft/ft

Stage ft	Elevation ft msl	Area × 1000 sq ft	Width ft	Manning's n
		1950 Flood Data		
20	19.48	140	3090	0.0201
25	24.48	155	3110	0.0192
30	29.48	170	3130	0.0183
32.5	31.98	178	3180	0.0180
35	34.48	185	3360	0.0168
40	39.48	201	3650	0.0143
45	44.48	220	3690	0.0133
47	46.48	226	3690	0.0129
		1973 Flood Data		
20	19.48	139	2330	0.0240
25	24.48	152	2380	0.0222
30	29.48	164	2490	0.0201
35	34.48	177	2820	0.0175
40	39.48	190	2880	0.0152
45	44.48	205	2920	0.0138
47	46.48	211	2930	0.0134

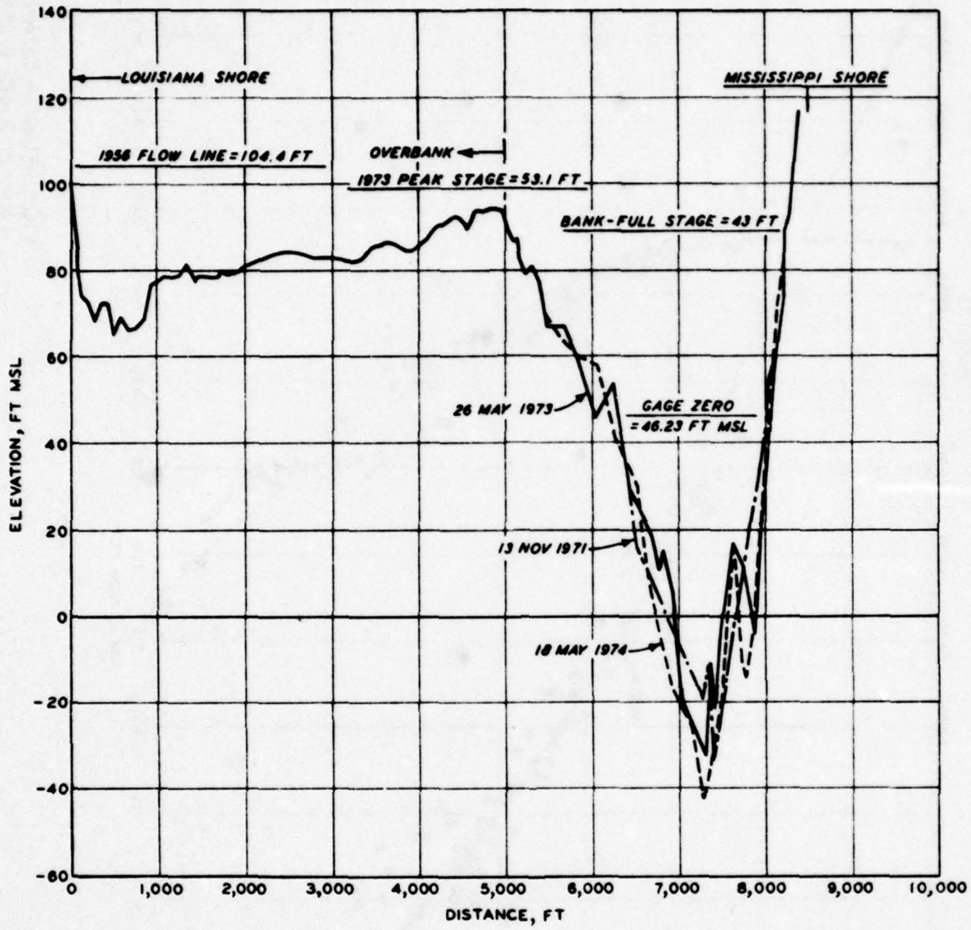
Table 6

Design Flood Hydrographs, Baton Rouge, La.

Daily Discharge, 1000 cfs

Project Design Flood (PDF) 58A-EN	
648	1500
666	Flow steady at 1500 for 63 days
690	
708	
723	
738	
760	
786	
817	
850	
883	
916	
941	1500
966	1450
995	1400
1028	1350
1062	1300
1100	1250
1142	1200
1178	1150
1210	1100
1239	1050
1269	1000
1298	950
1327	900
1348	850
1374	800
1395	750
1416	
1437	
1458	
1479	

1973/58A-EN		
648	1502	1347
666	1409	1336
690	1513	1247
708	1441	1325
723	1520	1258
738	1460	1214
760	1459	1180
786	1403	1225
817	1471	1158
850	1432	1136
883	1411	1125
916	1439	1114
914	1425	1111
966	1443	1112
995	1465	1125
1028	1479	1114
1062	1460	1081
1100	1491	1078
1142	1470	1085
1178	1489	1058
1210	1538	1040
1239	1484	1030
1269	1454	1051
1298	1441	1031
1327	1410	1023
1348	1384	978
1374	1453	958
1395	1448	945
1416	1445	927
1437	1414	892
1458	1414	898
1479	1425	724
1500	1381	
1383	1392	
1356	1347	



VICKSBURG–CROSS SECTION
AT DISCHARGE RANGE
RIVER MILE 435.24

PLATE 1

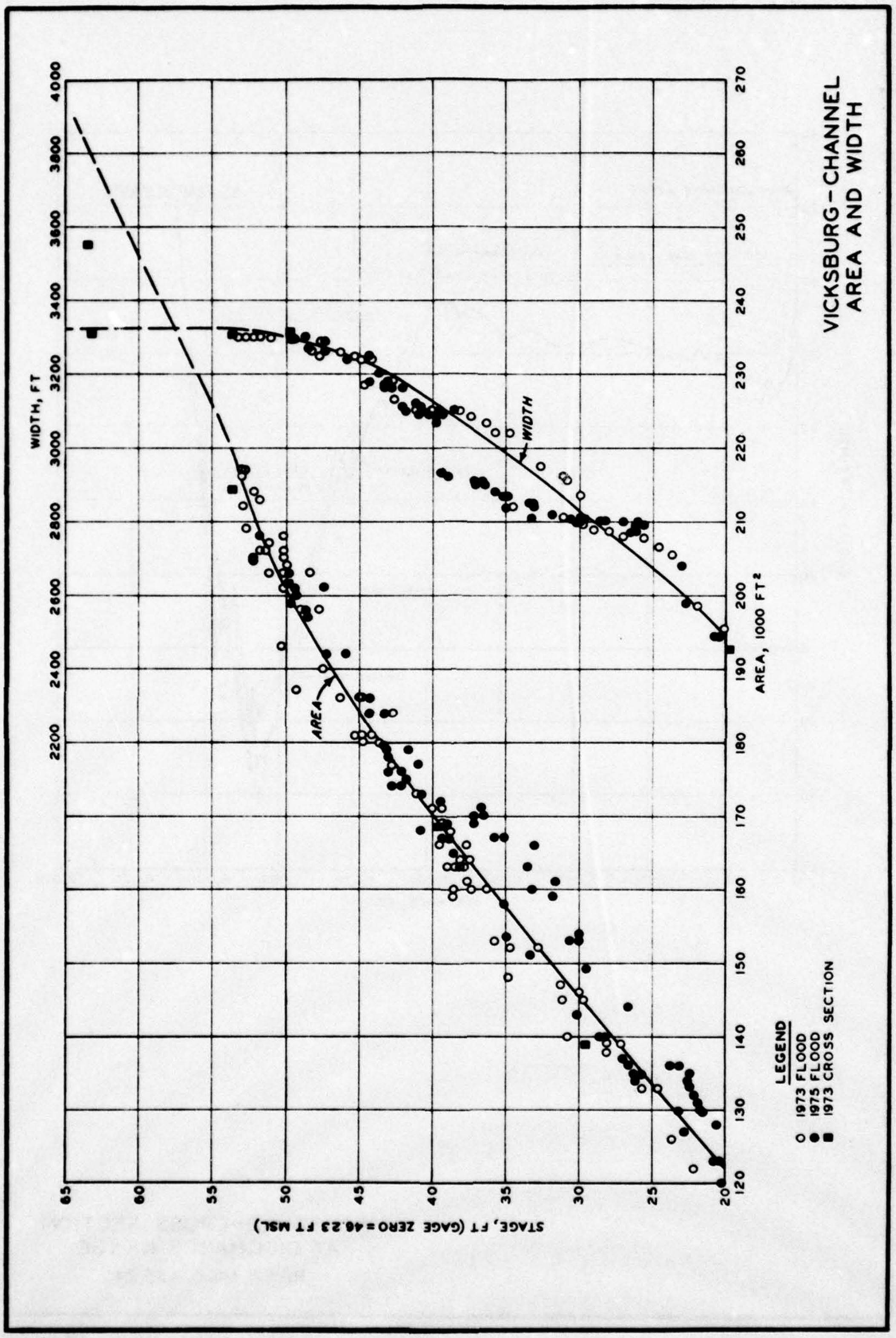
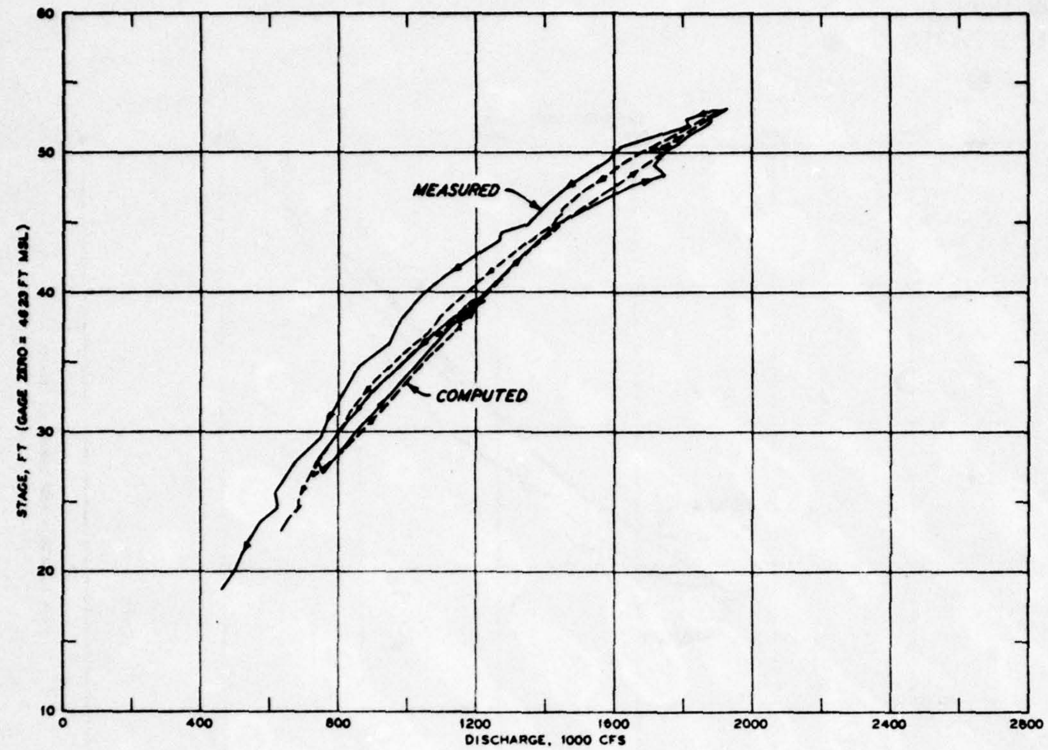
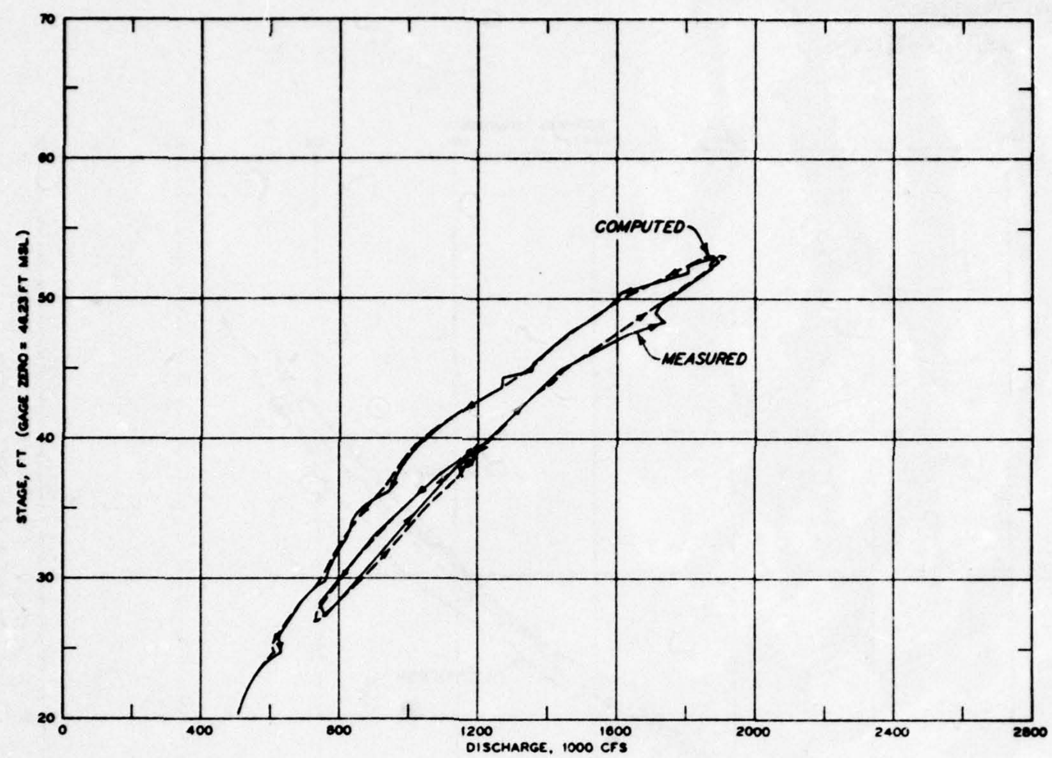

VICKSBURG - CHANNEL
AREA AND WIDTH
WIDTH, FT
AREA, 1000 FT²
STAGE, FT (GAGE ZERO = 46.23 FT MSL)
WIDTH
AREA
LEGEND
1973 FLOOD
1975 FLOOD
1973 CROSS SECTION

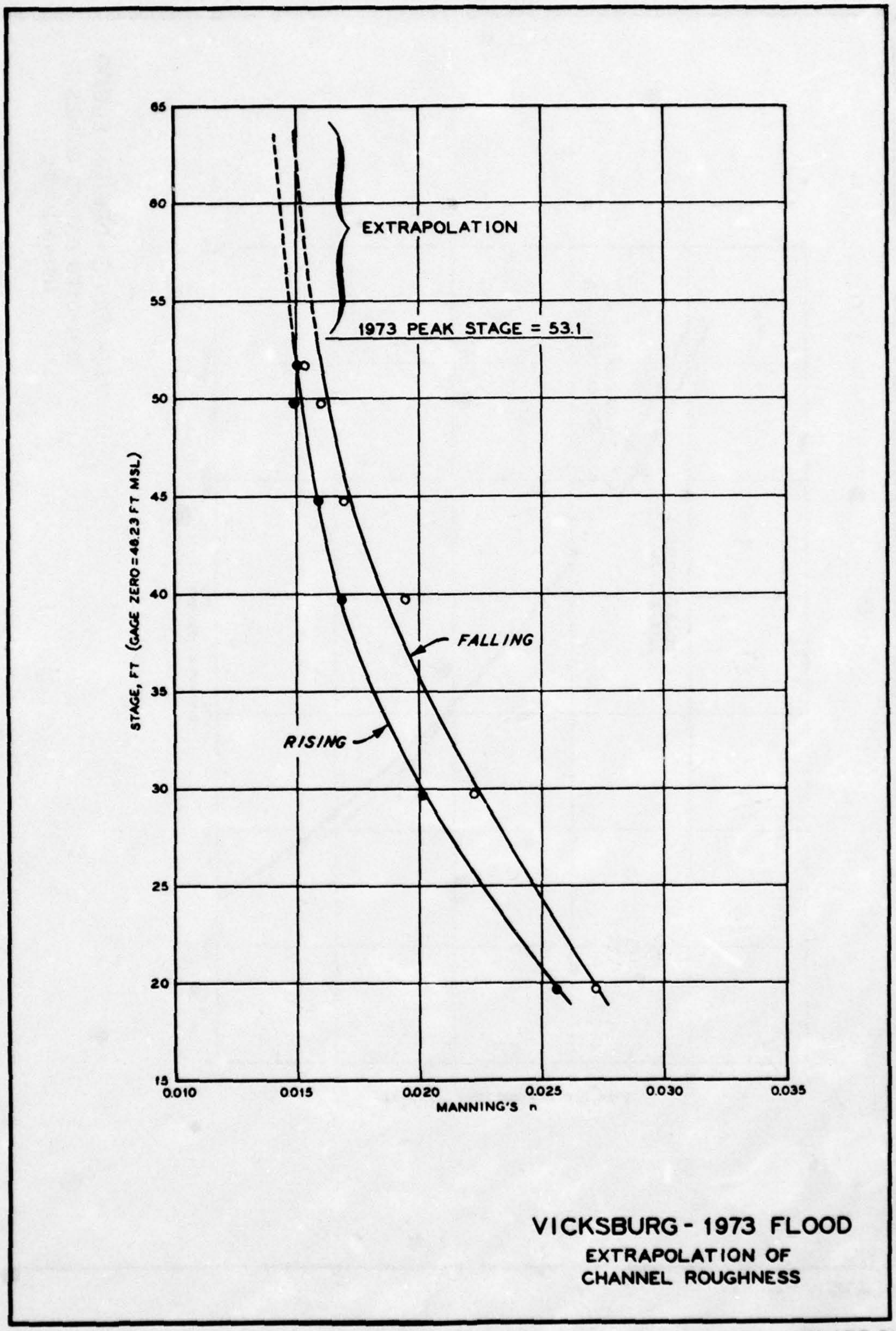
PLATE 2

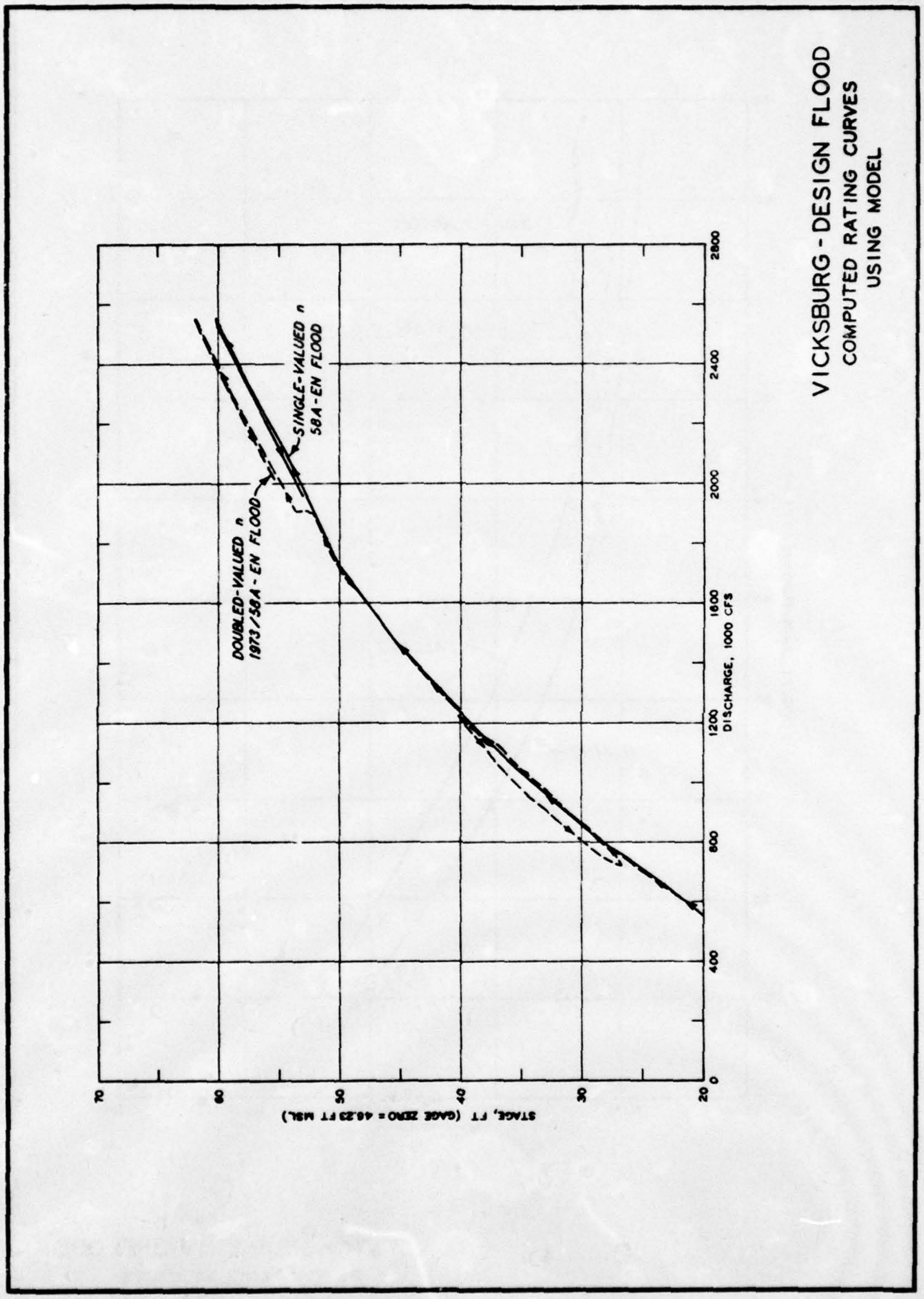


VICKSBURG - 1973 FLOOD
COMPUTED RATING CURVE
USING SINGLE-VALUED
MANNING'S n IN MODEL

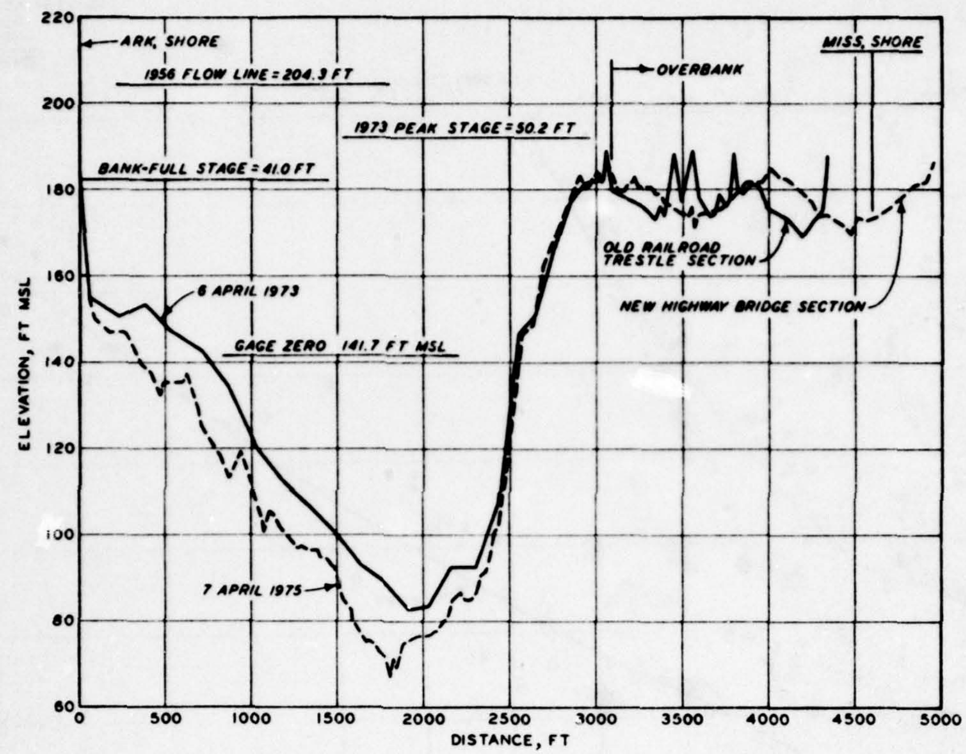


VICKSBURG - 1973 FLOOD
COMPUTED RATING CURVE
USING DOUBLE-VALUED
MANNING'S n IN MODEL


EXTRAPOLATION
1973 PEAK STAGE = 53.1
FALLING
RISING
STAGE, FT (GAGE ZERO=46.23 FT MSL)
MANNING'S n
65
60
55
50
45
40
35
30
25
20
15
0.010
0.015
0.020
0.025
0.030
0.035
VICKSBURG - 1973 FLOOD
EXTRAPOLATION OF
CHANNEL ROUGHNESS

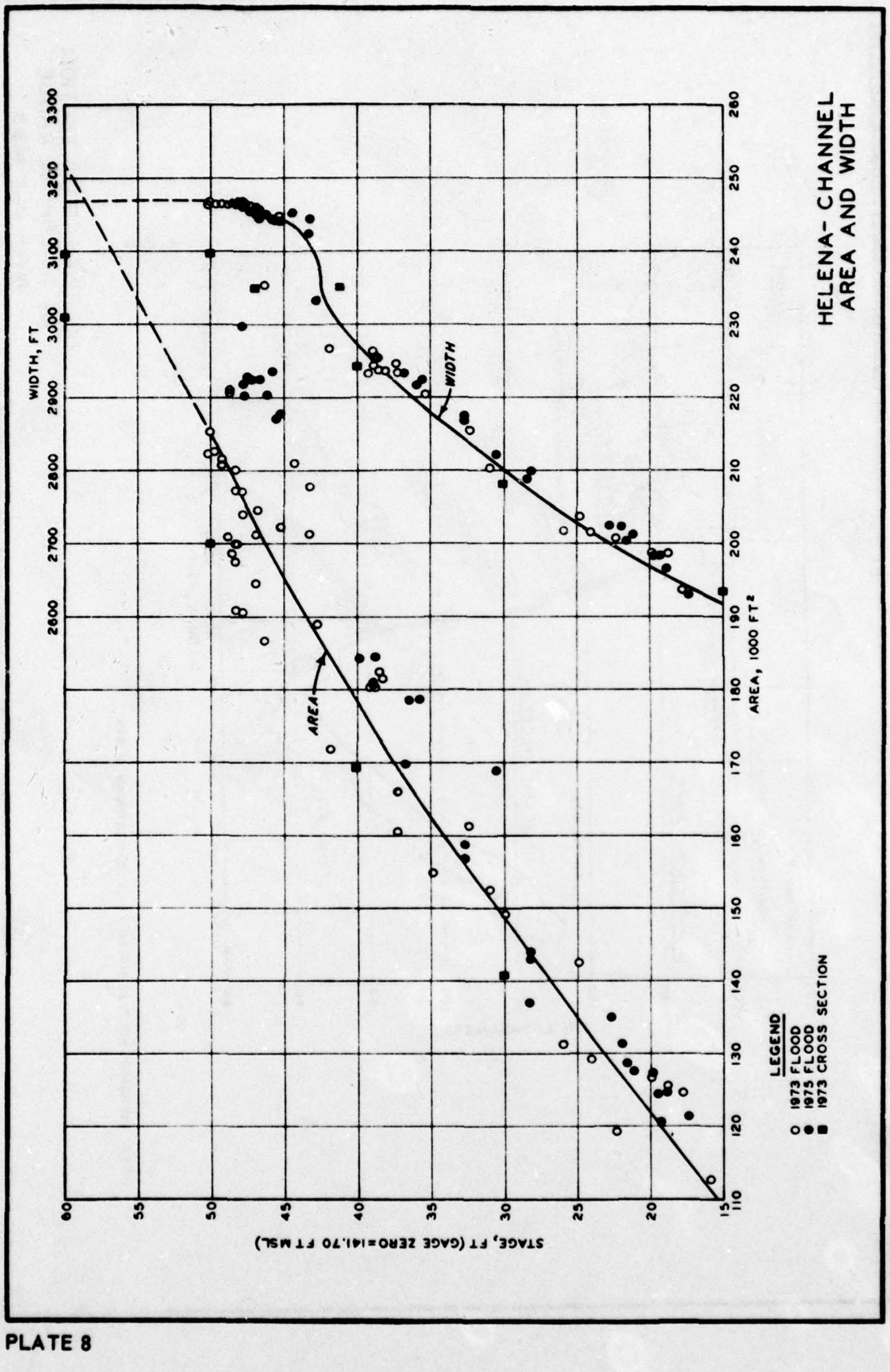


VICKSBURG - DESIGN FLOOD
COMPUTED RATING CURVES
USING MODEL



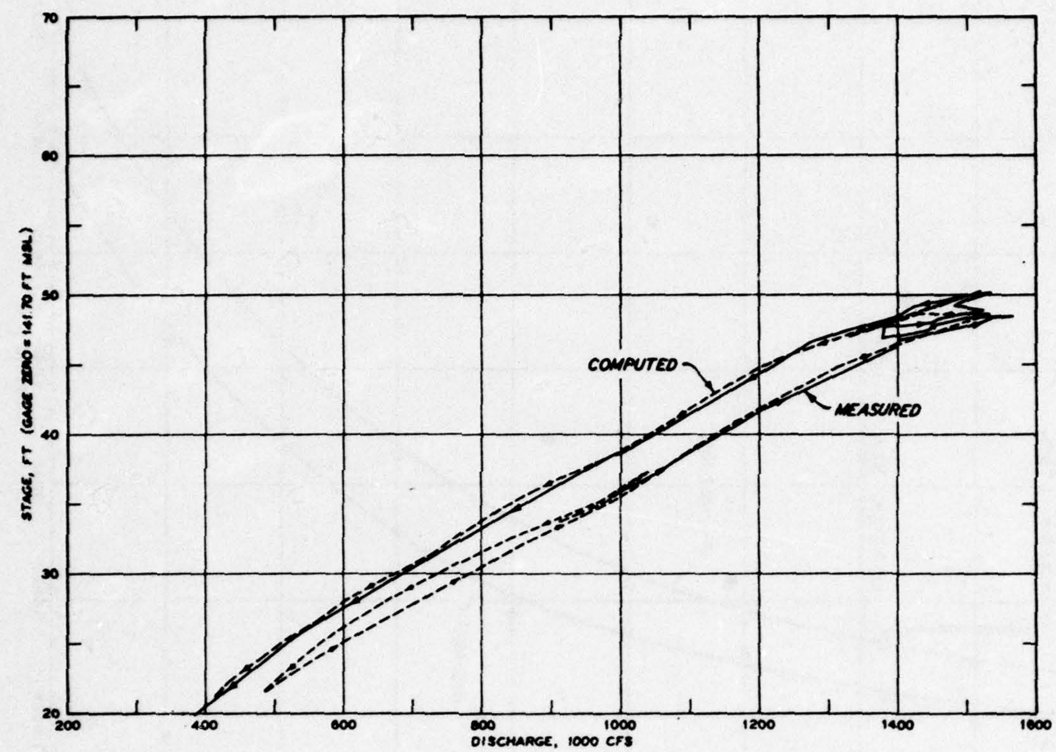
NOTE: OVERBANK SECTION MOVED 1 MILE DOWNSTREAM IN 1975.

HELENA-CROSS SECTION
AT DISCHARGE RANGE
RIVER MILE 663.3

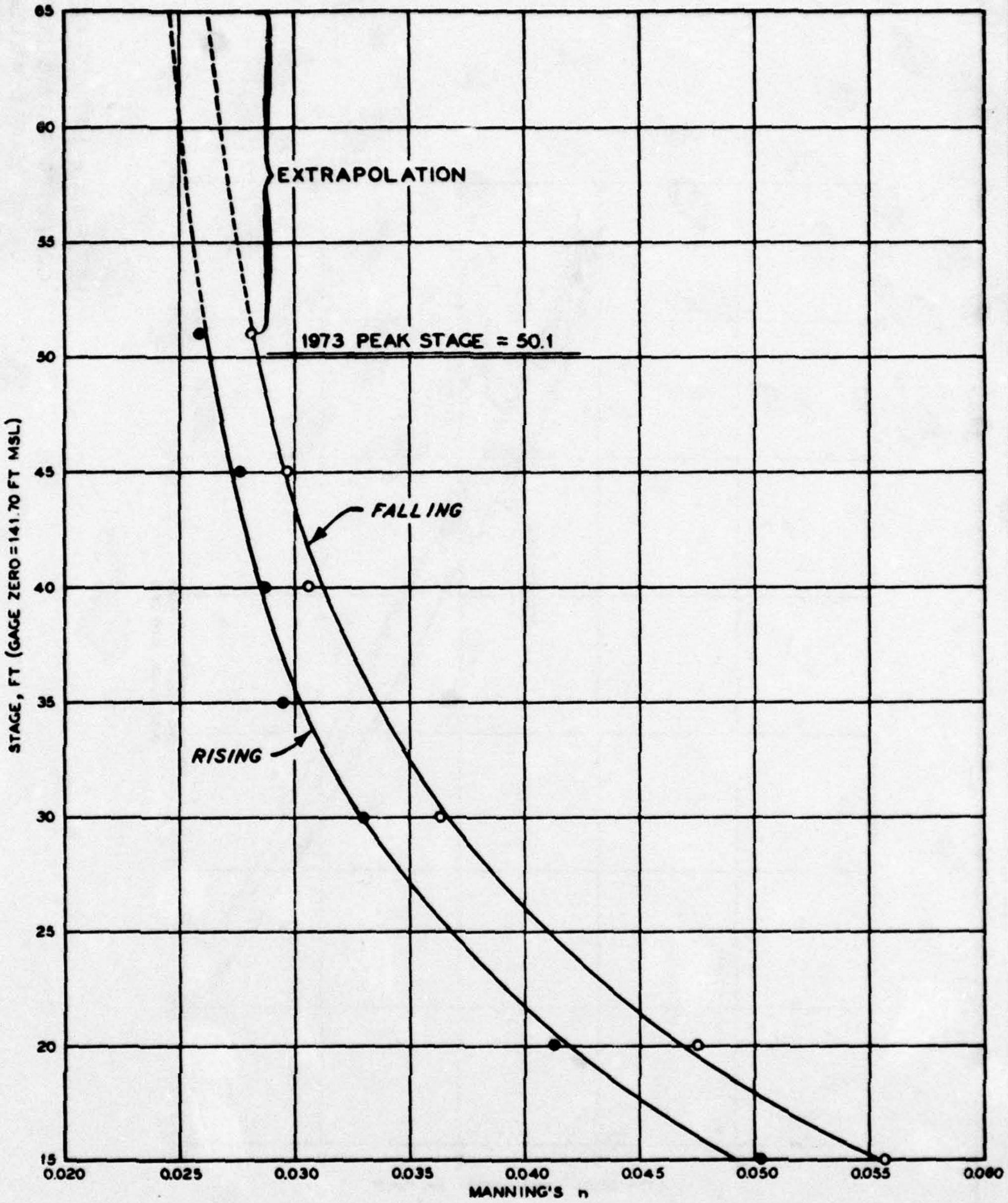


HELENA- CHANNEL
AREA AND WIDTH
WIDTH, FT
AREA, 1000 FT²
STAGE, FT (GAGE ZERO=141.70 FT MSL)
WIDTH
AREA
LEGEND
1973 FLOOD
1975 FLOOD
1973 CROSS SECTION

PLATE 8

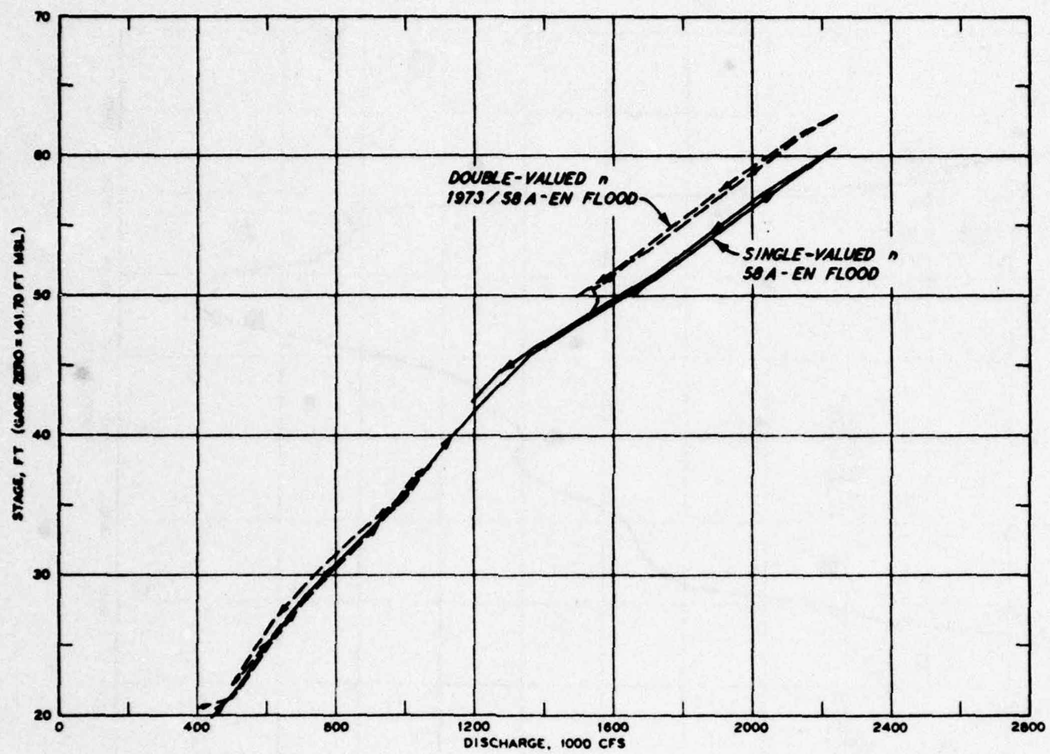


HELENA - 1973 FLOOD
COMPUTED RATING CURVE
USING DOUBLE-VALUED
MANNING'S n IN MODEL

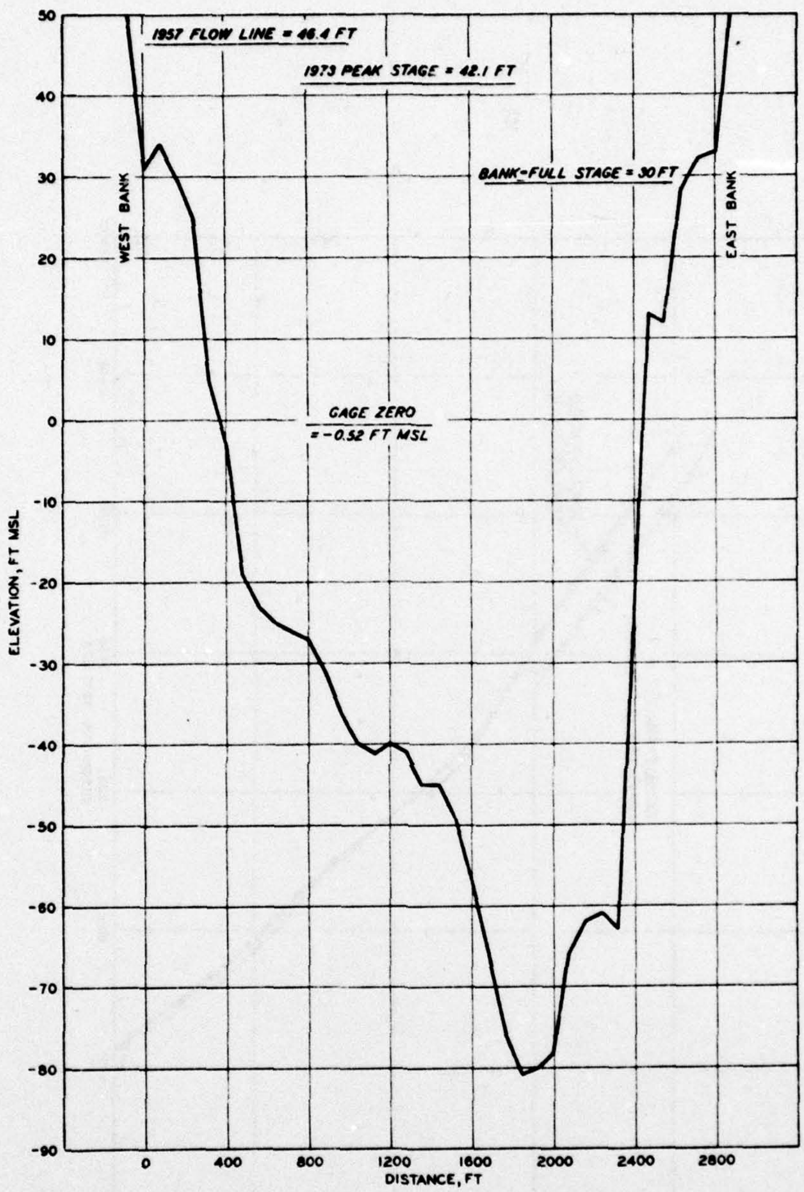


HELENA - 1973 FLOOD
EXTRAPOLATION OF
CHANNEL ROUGHNESS

PLATE 10



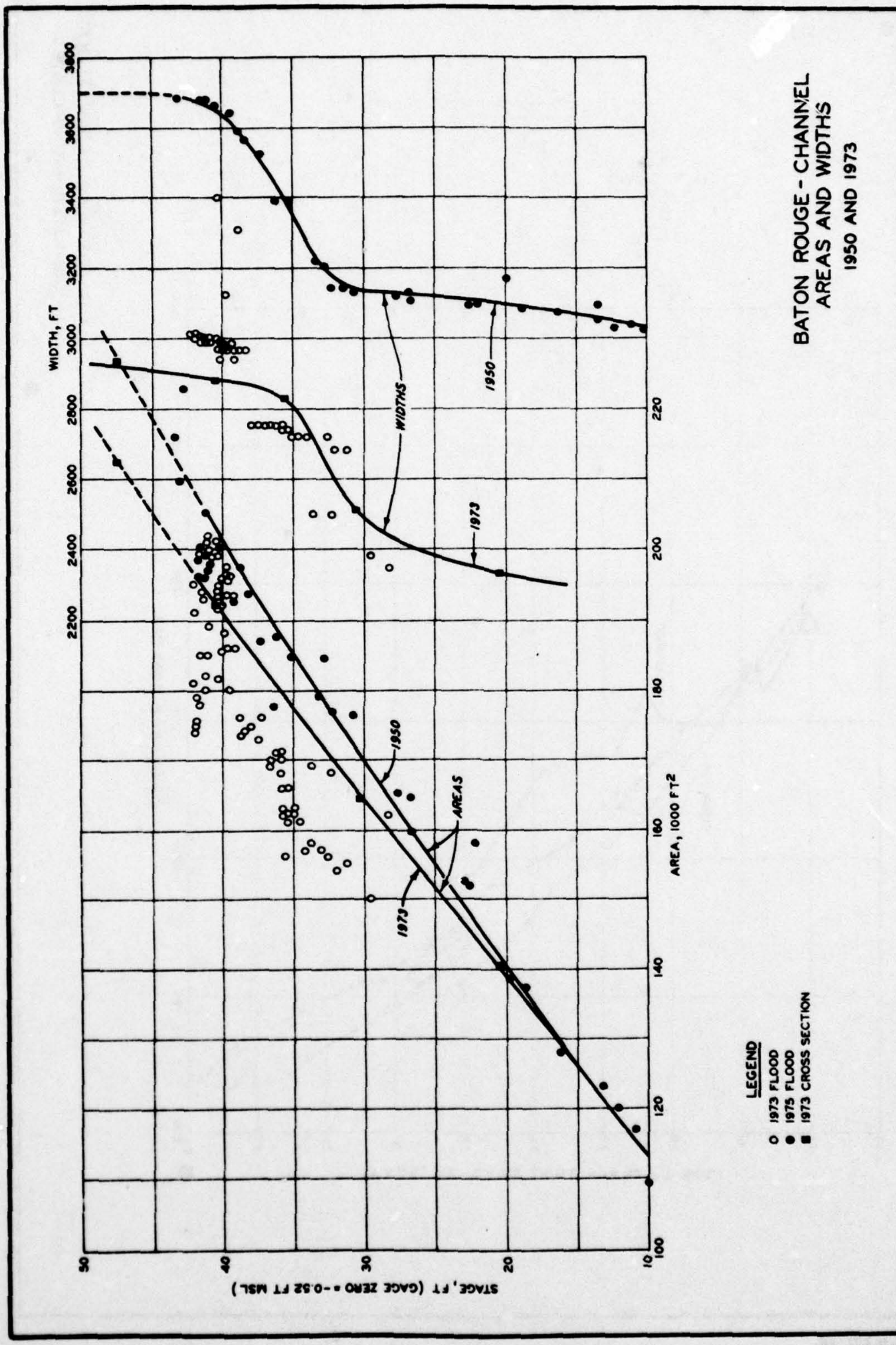
HELENA-DESIGN FLOOD
COMPUTED RATING CURVES
USING MODEL



BATON ROUGE - CROSS SECTION
AT DISCHARGE RANGE

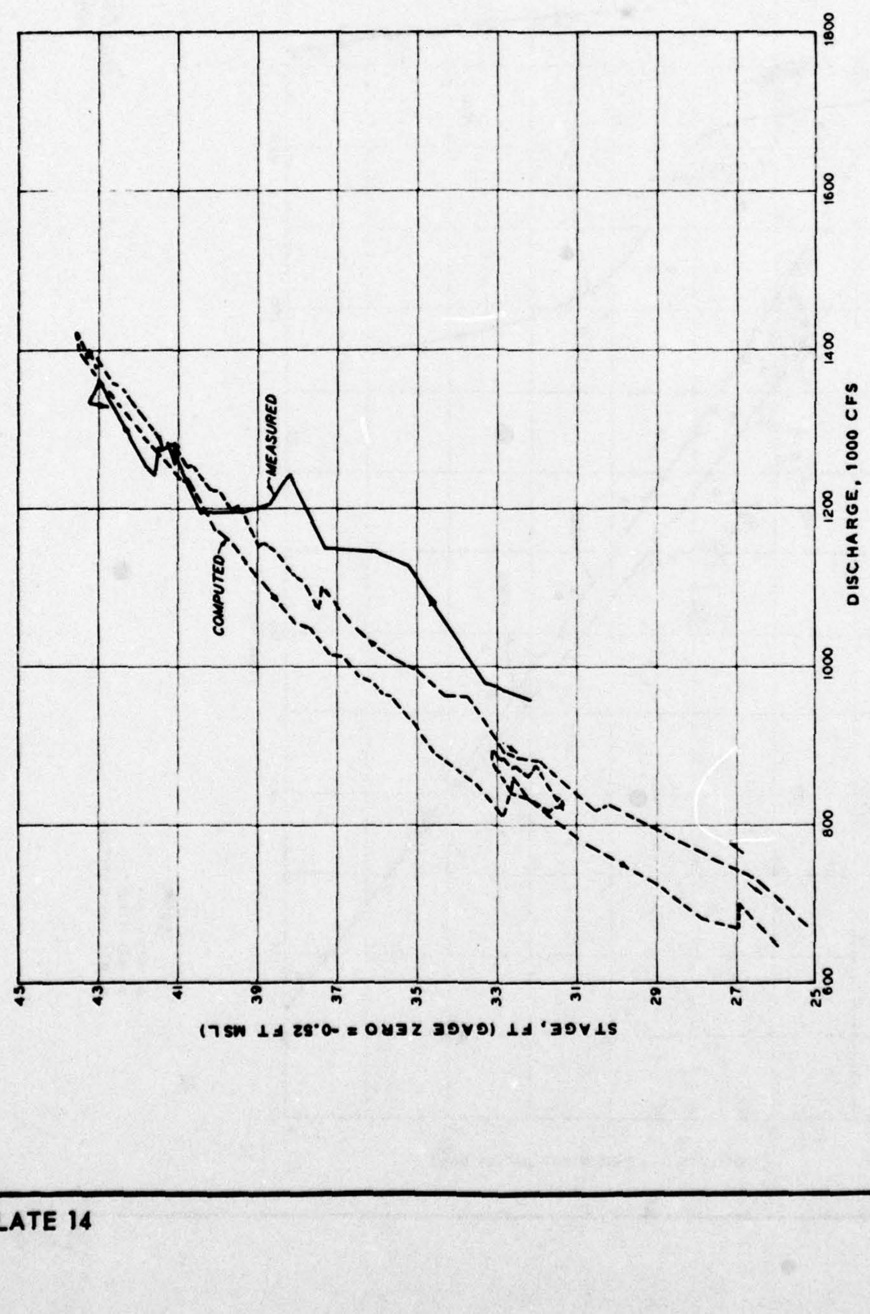
RIVER MILE 233.75
SURVEY 30 JAN 1974

PLATE 12



BATON ROUGE - CHANNEL AREAS AND WIDTHS
1950 AND 1973

PLATE 13



BATON ROUGE - 1950 FLOOD
COMPUTED RATING CURVES

PLATE 14

BATON ROUGE - 1973 FLOOD
COMPUTED RATING CURVES

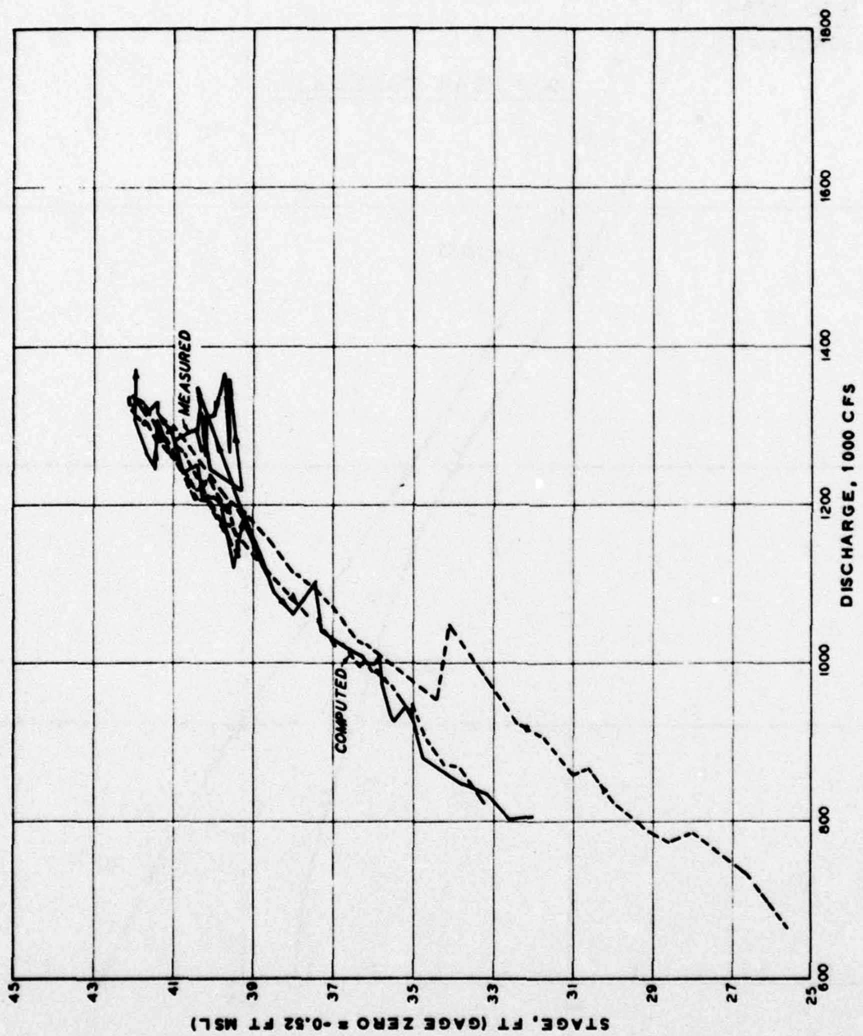
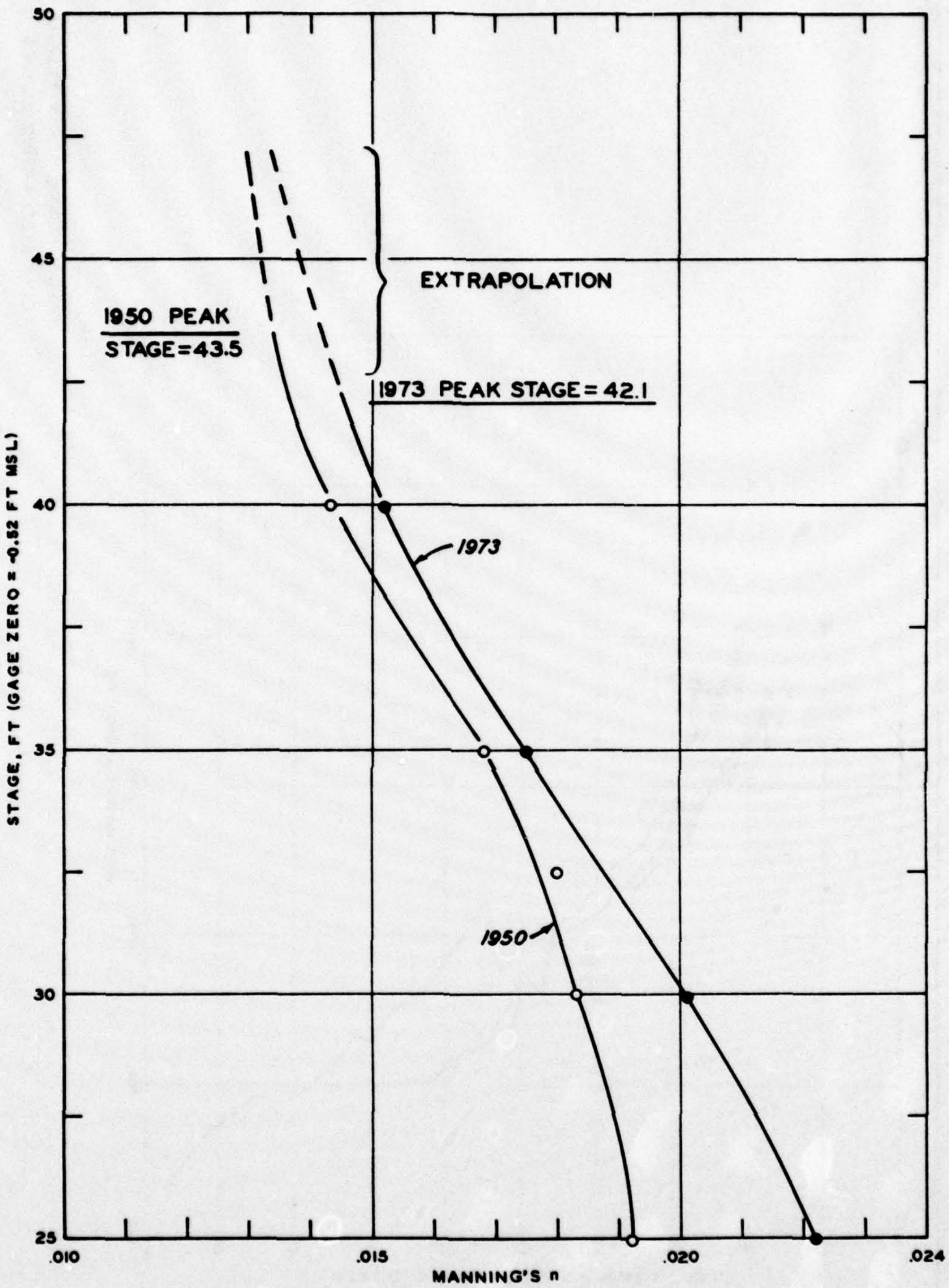
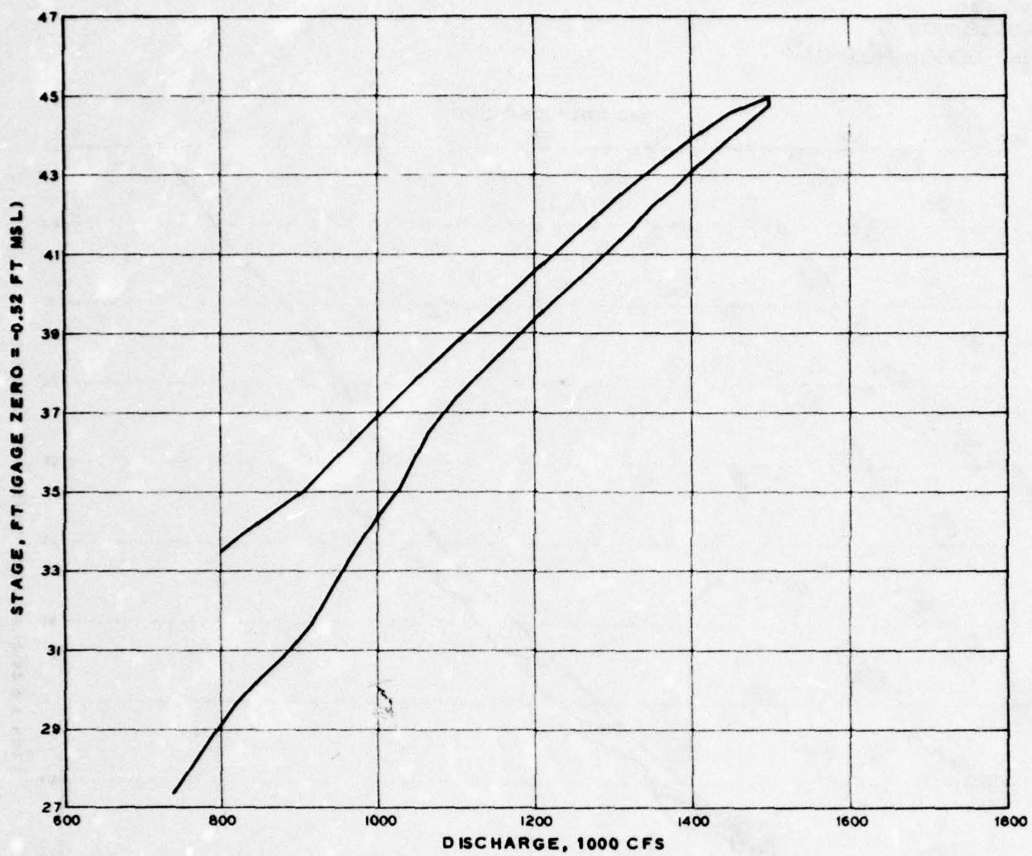


PLATE 15

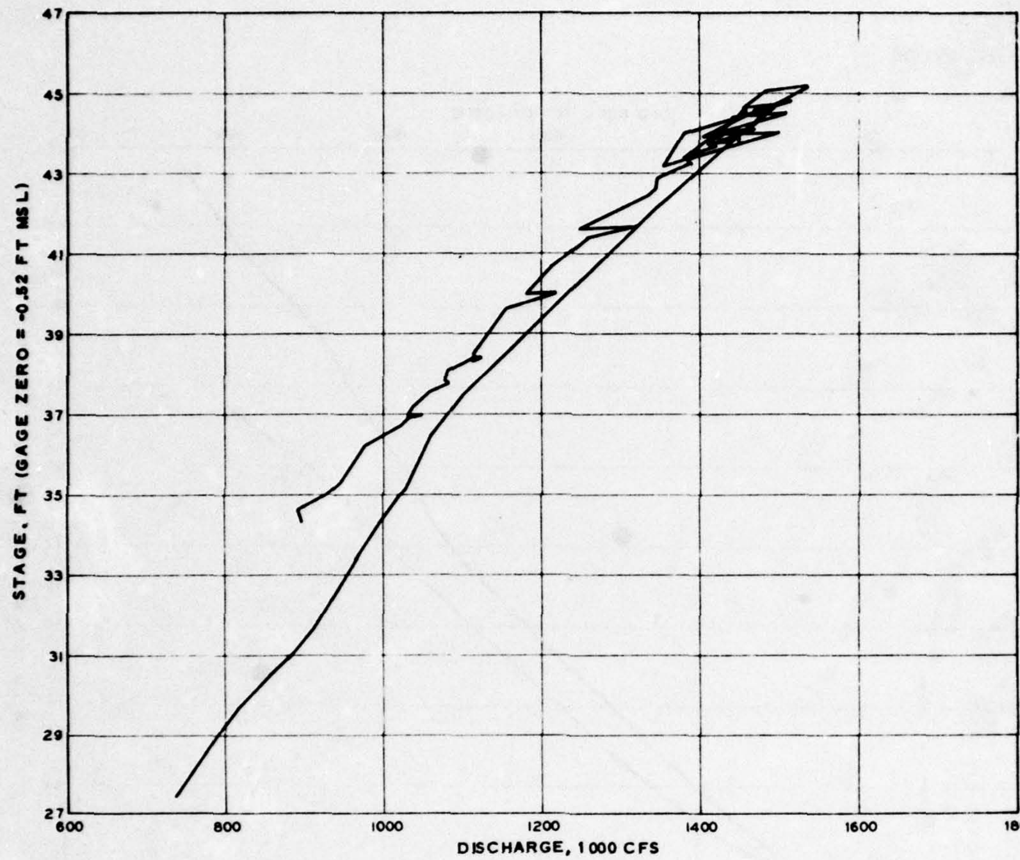


BATON ROUGE - 1950 AND 1973 FLOODS
EXTRAPOLATION OF CHANNEL ROUGHNESS

PLATE 16



BATON ROUGE - DESIGN FLOOD 58A-EN
COMPUTED RATING CURVE USING
1973 DATA CALIBRATION



BATON ROUGE - DESIGN FLOOD 1973/58A-EN
COMPUTED RATING CURVE USING
1973 DATA CALIBRATION

PLATE 18

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

Huval, Carl John

The dynamic loop effect on the Mississippi River project design flood flow line / by Carl J. Huval. Vicksburg, Miss. : U. S. Waterways Experiment Station ; Springfield, Va. : available from National Technical Information Service, 1979.

24, [6] p., 18 leaves of plates : ill. ; 27 cm. (Miscellaneous paper - U. S. Army Engineer Waterways Experiment Station ; HL-79-2)

Prepared for U. S. Army Engineer Division, Lower Mississippi Valley, Vicksburg, Mississippi.

References: p. 24.

1. Design flood. 2. Dynamic loop effect. 3. Flood control. 4. Flow characteristics. 5. Mathematical models. 6. Peak floods. 7. Mississippi River. 8. Unsteady flow. I. United States. Army. Corps of Engineers. Lower Mississippi Valley Division. II. Series: United States. Waterways Experiment Station, Vicksburg, Miss. Miscellaneous paper ; HL-79-2.

TA7.W34m no.HL-79-2